Robustness-based optimal pump design and scheduling for water distribution systems

Donghwi Jung, Kevin E. Lansey, Young Hwan Choi and Joong Hoon Kim

ABSTRACT

We introduce a new system robustness index for optimizing the pump design and operation of water distribution systems. Here, robustness is defined as a system's ability to continue functioning under varying demand conditions. The maximum difference between the daily maximum and minimum pressures of a node was taken as a robustness indicator and incorporated as a constraint in a pump design and operation model that minimizes the total pump cost (construction and operation cost). Two well-known benchmark networks, the Apulian and Net2 networks, were modified and used to demonstrate the proposed model. The Pareto relationship between the total cost and system robustness was explored through independent optimizations of the model for different robustness constraint values. The resulting solutions were compared to the traditional least-cost solution. Regardless of the study networks, considering the robustness resulted in a greater number of small pumps compared with the least-cost solution. A sensitivity analysis on tank capacity was performed with the Apulian network. The proposed model is the pump design and operation tool that accounts for both the total pump cost and system robustness, which are the most important factors considered by water distribution operators.

Key words | daily maximum pressure difference, pump design, pump operation, robust operation, water distribution network

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INTRODUCTION

Optimizing the operation of a water distribution system (WDS) is a complex task of determining the most cost-efficient set of states of control components (e.g., pumps and valves) during a predefined period. In the last two decades, the total energy cost has been considered to optimize the pump operation under various constraints: the system's performance under multiple demand conditions (Shamir 1974), the number of pump switches during the day (Lansey & Awumah 1994), substance concentrations that affect water quality (Sakarya & Mays 2000; Ostfeld & Salomons 2006), peak power use (McCormick & Powell 2003), and reliability and water quality (Farmani *et al.* 2006). Various mathematical and heuristic techniques have been used to optimize the pump operation (Zessler & Shamir 1989; Jowitt & Germanopoulos 1992; Ormsbee & Reddy 1995; Nitivattananon *et al.* 1996; van Zyl *et al.* 2004; Price & Ostfeld 2013; Kurek & Ostfeld 2014). A pump design for determining the number and capacity of pumping units has rarely been considered simultaneously with optimal pump operation (Ostfeld & Tubaltzev 2008).

Recently, methodologies for real-time pump scheduling have been proposed to lessen the computational burden from iterative optimizations where the predicted demand is updated every time interval (e.g., 1 h). Such methodologies include an artificial neural network (ANN)-based method (Jamieson *et al.* 2007; Martinez *et al.* 2007; Rao & Salomons 2007; Salomons *et al.* 2007; Wu & Behandish 2012a, 2012b), simplification or linearization of the full network (Shamir & Salomons 2008; Pasha & Lansey 2009,

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2010, 2014; Giacomello *et al.* 2013; Jung *et al.* 2014a), and a multi-algorithm approach (Odan *et al.* 2015).

Most system operators want to supply the required amount of water to their customers even under disturbances (e.g., pipe failure) and thus often place more importance on the system reliability than the operation cost. For example, most system operators want to keep the water level in a tank as high as possible in preparation for emergency conditions (e.g., firefighting). However, this requires constant pump operation, which results in high energy costs. Few studies have attempted to consider system reliability in WDS operation. Farmani *et al.* (2006) tried to address the concerns of system operators by considering the surplus power as a percentage of the net input power to be a system reliability indicator, while Odan *et al.* (2015) used a modified form of such an indicator; however, this indicator is not appropriate for achieving operational reliability.

Jung *et al.* (2014b) proposed a new robustness index for WDS design. They defined robustness as a system's ability to maintain its function under system disturbances and calculated it from the coefficient of variation of stochastic pressures under uncertain demands and pipe roughness. They applied the proposed robustness index to the multi-objective optimal design of the Anytown network (pipe and pump design) and compared the results with the solutions obtained from a traditional reliability-based design. They found that a robustness-based design, in contrast to a reliability-based design, can limit the variation in pressures and result in a smaller depth of failure (degree of system degradation) in the event of a pipe break and fire flow. A similar robustness index can be developed for operational reliability or robustness.

The impact of having many pumps on the system reliability has been discussed in some papers. Zhuang *et al.* (2013) introduced an adaptive pump and valve operation methodology (non-optimization-based method) to mitigate the hydraulic effect of pipe failures in a WDS. Their method identifies the best set of pumping units with various capacities after isolating the affected subsection of the system. Zhuang *et al.* concluded that having multiple pumps of various capacities increases the system water availability under failure conditions. This conclusion was also summarized by Lansey (2012), who described multiple pumps as an example of system redundancy.

Although most studies, with the exception of Ostfeld & Tubaltzev (2008), have solely considered pump operations under the assumption that the pump station configuration is already fixed, pump operations are significantly affected by the number of pumps and their capacity. A system's optimal pump operation can only be determined when the pump design and operation problem are formulated and optimized simultaneously.

In this study, a new robustness index was developed for optimal pump design and operation. Here, robustness was defined as the system's ability to maintain pressures within a given range under varying demand conditions. The maximum nodal difference between the daily maximum and minimum pressures was taken as the robustness indicator and incorporated as a constraint in the pump design and operation model to minimize the total pump cost (construction and operation cost). Two well-known benchmark networks, the Apulian and Net2 networks, were modified and used to demonstrate the proposed model. The decision variables were the number of pumping units in a pump station downstream of a single fixed source, the capacity of the system, and the control statuses during the scheduling period. Different robustness levels were considered for the robustness-based optimizations independently, and the resulting solutions were compared to the traditional least-cost solution. In order to investigate the impact of changes to the tank capacity on the pump design and operation, an Apulian network with a bigger tank was compared against the Apulian network with the original tank.

METHODOLOGY

The proposed model's objective is to minimize the total cost of the pump construction and operation. The system robustness is measured according to the maximum difference between the daily maximum and minimum pressures of a node and incorporated as a constraint in the model. The pump design involves determining the optimal number and capacity (e.g., kW) of pumping units in a pump station. For optimal pump scheduling, the pumping unit's status (ON/OFF) is determined each hour for 24 h. Note that this study focused on finding the optimal diurnal pump operation, which would be used as a reference and repeated each day under real-life conditions.

The following sections describe the details of the objective functions and optimization approach.

Pump construction cost

The pump construction cost (PCC) is a function of many properties of the facility, including the cost of local labor and materials, the type of pump (variable or constant speed), the sophistication of supervisory control and data acquisition (SCADA) equipment, and other site-specific conditions (Walski 2012). Walski (2012) suggested using a general power function that relates the PCC to the rated flow (QP) and head (HP). The parameter a to be multiplied with the function varies based on the properties of the facility (e.g., presence of a standby generator, site-specific conditions), while the exponent $b = 0.7$ is suggested for a rated flow, and $c = 0.4$ is recommended for the rated head in the power function ($\mathrm{PCC} = aQP^bHP^c$). Therefore, the PCC is very sensitive to the flow. We employed the following PCC equation, which was introduced by Walski et al. (1987):

$$\mathrm{PCC} = npump \times 500 \times QP^{0.7} \times HP^{0.4} \tag{1}$$

where $npump$ is the number of pumping units to be installed, QP is the rated discharge (gpm) per pumping unit, which is equal to the total system design demand divided by $npump$, and HP is the rated head (ft) through the pumping units.

Pump operation cost

In a pump station, electricity is required not only to run the pumping units but also to operate the system control and monitoring systems (e.g., SCADA). Unit energy prices generally vary throughout the day because of the power utility and government policies to incentivize customers who avoid electricity usage during periods of the day with high energy costs. The pump operation cost (POC) considers the electricity tariff structure as follows:

$$\mathrm{POC} = 365 \times PVF \times \frac{\phi}{\eta} \times \sum_{j=1}^{npump} \left(\sum_{t=1}^{T} EC_t H_{jt} Q_{jt} \right) \tag{2}$$

where PVF is the present value factor $\left(= \frac{(1+AI)^{pp} - 1}{AI(1+AI)^{pp}} \right)$, AI is the annual discount rate, pp is the planning period in years, ϕ is the power coefficient unit conversion factor ($0.746/270\ \mathrm{kW} \times \mathrm{h/m^4}$), η is the pump efficiency, t is the time step index ($t = 1, 2, \ldots, 24$), EC_t is the energy cost at the time step t (USD/kWh), H_{jt} is the head gained by pumping unit j at the time step t (m), and Q_{jt} is the flow supplied by pumping unit j at the time step t ($\mathrm{m^3/h}$).

System robustness indicator

Jung et al. (2014a, 2014b) minimized the total cost and maximized the system robustness in their pipe and pump design for a WDS. They defined the robustness as a system's ability to limit the failure depth in the event of disturbances such as uncertain demand and changes in the pipe roughness. The nodal robustness is calculated from the coefficient of variation of stochastic pressures at the node, and the system robustness is formulated as the minimum nodal robustness. Therefore, the robustness-based WDS design problem is a min-max/max-min problem (i.e., minimizing the maximum or maximizing the minimum component's measure). Min-max and max-min formulations are common in robustness-related engineering optimizations because they avoid the worst-case scenario (Krause et al. 2008).

Highly variable and excessive pressure should be avoided for reliable system operation and leakage management. In this study, the proposed model limits the range of pressures throughout 24 h (i.e., 1 day) at the critical node. The robustness at the ith node (NRob_i) is defined as the daily maximum pressure difference, which is calculated as:

$$\mathrm{NRob}_i = \max Pres_i - \min Pres_i \tag{3}$$

where $\max Pres_i$ is the daily maximum pressure at the ith node and $\min Pres_i$ is the daily minimum pressure at the ith node. Then, the system robustness (SRob) is defined as the maximum nodal robustness and is calculated as follows:

$$\mathrm{SRob} = \max (NRob_i) \qquad i = 1, \ \ldots, \ n \tag{4}$$

Optimal pump design and operation model for WDS

A water system operator wants to minimize the economic cost for pump construction and operation while keeping the pressures and system reliability sufficiently high. The proposed model minimizes the total sum of the pump construction and operation costs with constraints on the pressures, tank level, and system robustness as follows:

$$\text{Minimize}\, F = \text{PCC} + \text{POC} \tag{5}$$

$$\text{s.t. SRob} \leq \text{SRob}_{\text{req}} \tag{6}$$

$$\text{Hpres}_i \geq \text{Hpres}_{\text{req}} \qquad i = 1, \ \ldots, \ n \tag{7}$$

$$\text{TL}_{\min} \leq \text{TL}_t \leq \text{TL}_{\max} \qquad t = 1, \ \ldots, \ T \tag{8}$$

$$\text{TL}_1 - 0.1 \leq \text{TL}_T \leq \text{TL}_1 + 0.1 \tag{9}$$

where SRob_{req} is the system robustness requirement, Hpres_i is the pressure at the ith node, $\text{Hpres}_{\text{req}}$ is the pressure requirement, $\text{TL}_{\min}$ and $\text{TL}_{\max}$ are the minimum and maximum water levels, respectively, in a tank, and TL_t is the tank water level at the time step t. The tank level at the final scheduling time step (TL_T) should be within ±0.1 m of the level at the initial scheduling time step (TL_1) in order to consider the continuity of the operation. Note that the WDS optimization problem is constrained by the conservations of mass and energy. Here, these constraints were implicitly satisfied through a hydraulic simulation performed on EPANET (Rossman 2000).

Genetic algorithm

The genetic algorithm (GA) was employed to optimize the pump design and operation. A GA consists of three operators: selection, crossover, and mutation. An even number of individual solutions are selected from a randomly generated initial population based on roulette wheel rules by which solutions of high fitness have a high probability of being selected. During the crossover, the selected solutions share their traits embedded in chromosomes. For example, the pump operations of two solutions during a period can be exchanged. While the crossover operator helps the algorithm explore the solution space, the mutation operator guarantees exploitation of the search and an escape from local optima. In the general GA, mutation alters one or more chromosomes of the selected solution from the original value. In this study, an initial population for the GA was randomly generated. Multipoint crossover occurred with 95% probability, while each chromosome value could be changed with a 5% probability. Therefore, a real random number was generated by each chromosome of the selected solutions, and if it was less than 0.05, mutation occurred. A penalty function approach was applied to naturally exclude solutions that did not satisfy any of the constraints. For example, a penalty factor of 5 million USD was considered to represent a deficiency in system robustness, and the product was added to the total cost.

SUMMARY OF ASSUMPTIONS

A number of assumptions and simplifications were made in this study: (1) the network hydraulic model perfectly represents a real system; (2) the pipe roughness of 130 is constant throughout the planning period of 20 years; (3) when a pump changes its status, it should maintain the status for at least 1 h; (4) the option to change the pump status is only available once every hour; (5) the pump efficiency is a constant 75% throughout the planning period; (6) a uniform normal day operation is assumed to continue during the planning period; and (7) the valves and other hydraulic components around and within the pump station are represented as and embedded within pump links in EPANET. More details and specific simulations of the components were outside the scope of this study.

STUDY NETWORK

Two networks were used to demonstrate the proposed pump design and operation model: the Apulian (Figure 1) and Net2 (Figure 2). The former is a looped network, while the latter is a branch-dominated network. The total system demand of the Apulian network is 282 L/s, while that of the Net2 network is 20.4 L/s.

First, we applied a deterministic optimal WDS design approach introduced by Giustolisi *et al.* (2009) to determine the pipe diameters of the original Apulian network. This approach was originally intended to provide an initial

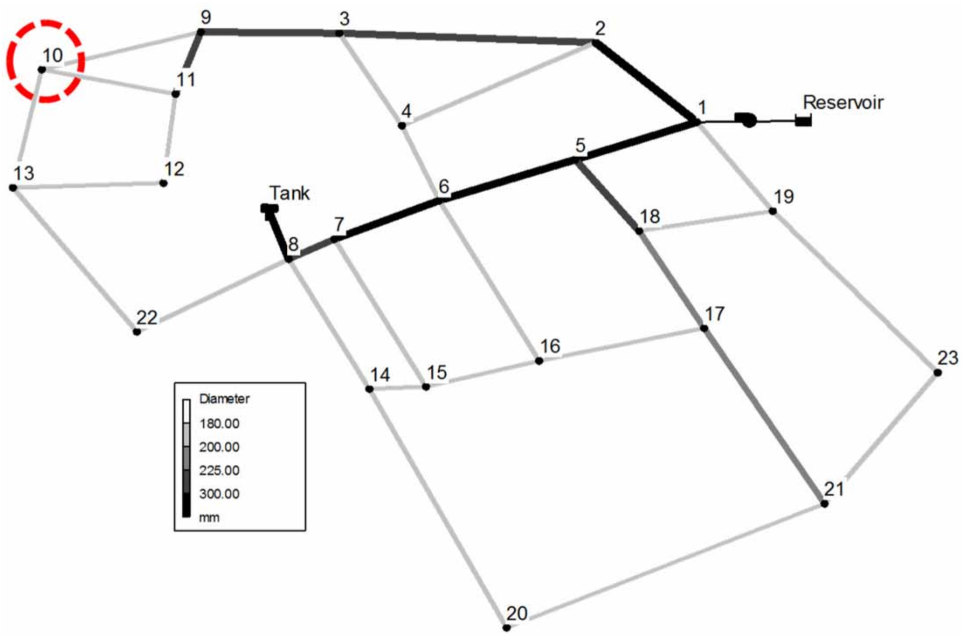

Figure 1 | Apulian network layout with node 10 circled (one of the most critical nodes).

population for multi-objective WDS optimization. Here, the pipe construction cost was minimized under a constraint on the pressure requirement (28 m). Commercial pipe diameters and the associated unit costs presented by Giustolisi *et al.* (2009) were considered. The resulting network had 300 mm pipes for the main lines passing through the middle of the two main loops (Figure 1). The optimized distribution pipes were between 100 and 180 mm.

Then, a cylindrical tank that could store about 4 h of the peak system demand was added to node 8 with a riser pipe that connected the tank to the system (Figure 1). The elevation difference between the maximum and minimum tank levels was 4 m, while two different diameters were considered for the optimization (described in more detail below). We assumed that the total fixed water head of a reservoir in the original Apulian network was lowered from 36 to 18 m. Thus, a pump station needed to be constructed at the link connecting the reservoir and node 1. All other information about the system (e.g., pipe length and topography) was adopted from Giustolisi *et al.* (2009). A Hazen–William roughness C factor of 130 was used for all pipes. Overall, the modified Apulian network had 23 nodes, 33 links, one reservoir, and one tank.

For the sensitivity analysis of tank capacity, two different tank capacities were considered for the modified Apulian network: a tank diameter of 35 m (Apulian-35) and a tank diameter of 52.5 m (Apulian-52.5). The maximum and minimum tank water levels remained the same. The total tank volume of the former was 3,779 tons, while that of the latter was 12,755 tons (238% larger than the former).

A few modifications were made to the original Net2 network, which is one of the example networks provided as part of the EPANET model. First, a reservoir with a total head of 75 m was added upstream of node 1, which models a reservoir in the original Net2 network (Figure 2). The negative demand due to it being a reservoir was removed from node 1. It was assumed that a pump station needed to be constructed at the link connecting the reservoir and node 1. All other information about the system (e.g., tank dimensions) remained the same as with the original Net2 network and can be found in Rossman (2000). The total tank volume of the Net2 network was 1,087 tons, and the elevation difference between the maximum and minimum tank levels was 6.1 m.

The decision variables were the number of pumping units in the pump station, their capacity in kilowatts, and the hourly pump status over 24 h. The pumps were assumed to be identical. In the Apulian network, the pump capacity was selected from the commercial pump capacities given in Table 1, while the corresponding pump characteristics

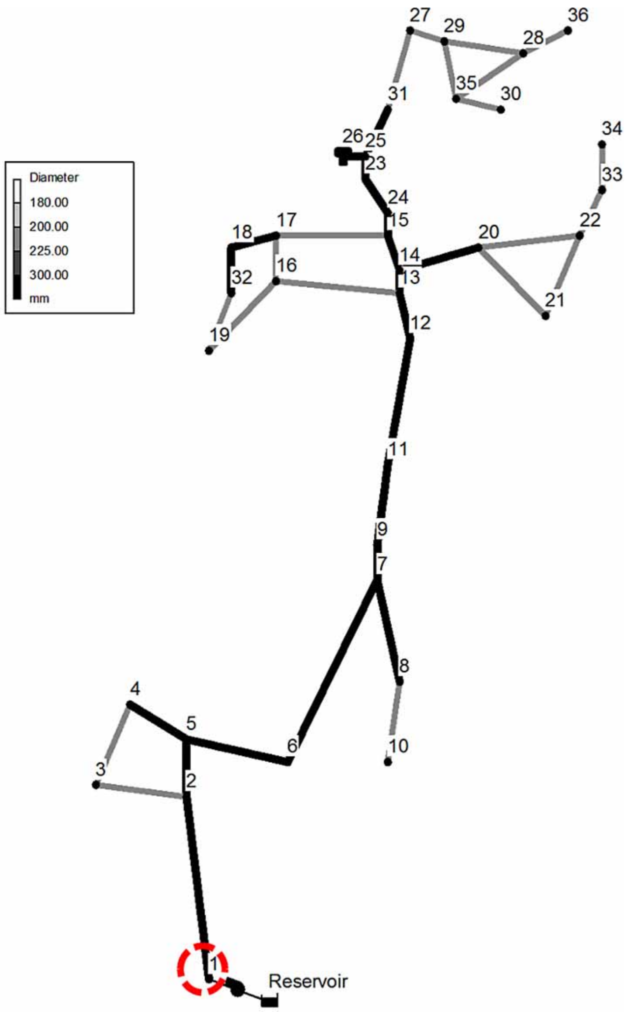

Figure 2 | Net2 network layout with a circle around node 1 (the most critical node).

Table 1 | Commercial pump capacities used for Apulian network pump optimization

Commercial pump ID	Design flow (L/s)	Design head (m)	Power (kW)
1	100	5	5
2	102	10	10
3	104	15	15
4	106	19	20
5	108	24	25
6	110	28	30
7	112	32	35
8	114	36	40
9	116	40	45
10	118	43	50
11	120	47	55
12	122	50	60
13	124	53	65
14	126	57	70
15	128	60	75
16	130	63	80
17	132	66	85
18	134	69	90
19	136	71	95
20	138	74	100

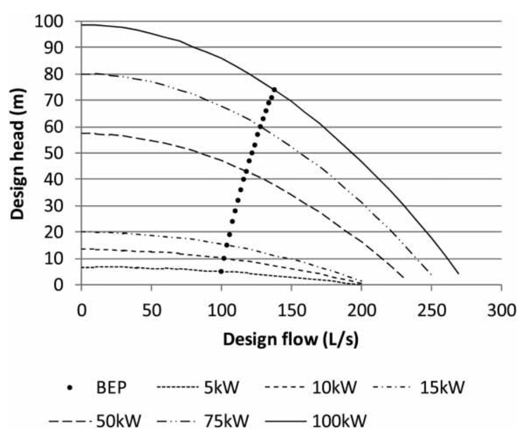

Figure 3 | Pump curves for commercial pumps given in Table 1.

curve is presented in Figure 3. The commercial pump capacities ranged from 0.1 to 5.2 kW in the Net2 network. A maximum of 10 pumping units could be installed in the pump station.

A diurnal water demand pattern was considered for the two networks, as shown in Figure 4. The daily maximum hourly demand had a peaking factor of 1.5 from 6 to 7 pm while the minimum hourly demand had a peaking factor of 0.6 at 2 am. A typical electricity tariff structure (Table 2) with three discrete periods was applied to both networks to calculate the pump energy cost. The electricity unit cost was 3.25 times higher during the peak hours than during the lowest tariff hours. Avoiding pumping during peak hours (6 am–3 pm) reduced energy costs; however, this was difficult to achieve because of the high water demand during this period. For the calculation of the POC, the optimal diurnal pump operation was assumed to be repeated over the planning period (20 years).

Hydraulic simulations were performed using EPANET. The pump status changes over 24 h were modeled with

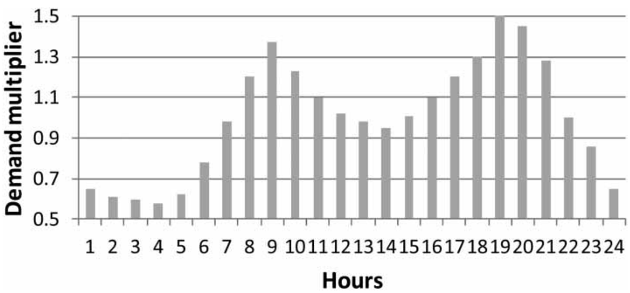

Figure 4 | Hourly demand pattern.

Table 2 | Energy tariff structure

Time	Energy cost (USD/kWh)
00:00–06:00	0.08
06:00–15:00	0.26
15:00–21:00	0.17
21:00–24:00	0.08

EPANET's extended period simulation and variable pump speed pattern. A pump speed factor of zero was considered to model the OFF status of the pump, while a speed factor of 1 was considered for the ON status.

APPLICATION RESULTS

This study investigated the impact of considering the operational system robustness when optimizing the pump design and operation of the case study networks. The total costs of the pump construction and operation were minimized by limiting the daily maximum pressure differences at the critical node to (1) 10, 9, 8, 7, and 6.6 m in the Apulian-35 network; (2) 15, 10, and 9 m in the Apulian-52.5 network; and (3) 1 and 0.7 m in the Net2 network. The resulting pump designs and operations were compared with the traditional pump design and scheduling obtained by minimizing the economic cost only. Other constraints on the minimum pressure requirement and tank levels (i.e., Equations (7)–(9)) were considered for each optimization. A water distribution network is more robust with small maximum pressure differences than with a large maximum pressure difference because the former indicates consistent performance against disturbances to the demand.

Economic cost results

In the Apulian-35 network, six independent optimizations were performed to obtain the optimal design and operation of different operational robustness levels. Pareto optimal solutions are plotted in Figure 5. As expected, the total cost increased as the required daily maximum pressure difference decreased (i.e., required operational robustness level increased). A sharp increase in the total cost was observed when the maximum pressure difference was less than 10 m (14.2 psi). The difference between the maximum and minimum pressures was about 25 m (35.5 psi) for the solution that did not consider robustness. Interestingly, only the PCC increased in the Apulian-35 network, while the POCs of the solutions stayed constant (Figure 5 and Table 3). Therefore, the proportion of the POC to the total cost decreased from 71% for the least-cost solution to 53% for the most robust solution, while that of the PCC increased from 29 to 47%. The POCs were around 4.2 million USD regardless of whether or not robustness was considered. However, the PCC of the most robust solution were about 2 million USD higher than the investment required by the least-cost solution (Table 3).

However, such changes in the proportions of the pump construction and operation cost were not observed in the Apulian-52.5 network (Table 4). Because the tank water level changes are not rapid, the highest robustness level (9 m) in the Apulian-52.5 network was bigger than that (6.6 m) in the Apulian-35 network. To compensate for the bigger tank size, a bigger pump was constructed for the Apulian-52.5 network than for the Apulian-35 network (e.g., two 90 kW pumps in the least-cost solution of the former and two 75 kW pumps in that of the latter). To have the same level of robustness, fewer total costs were invested in the Apulian-52.5 network than in the Apulian-35 network (e.g., 6.5 million USD for $SRob_{req}$ = 9 m in the former, while 6.9 million USD for the same $SRob_{req}$ of the latter) (Tables 3 and 4). The improved resourcefulness from the increased tank capacity helped improve the cost-effectiveness of the robustness-based operation.

The total system demand of the Net2 network was 7.2% of that of the Apulian networks, while the original reservoir head of both networks was lowered by about 20 m. Therefore, the commercial pump capacities considered in the Net2 network were smaller, and their pump curve showed less

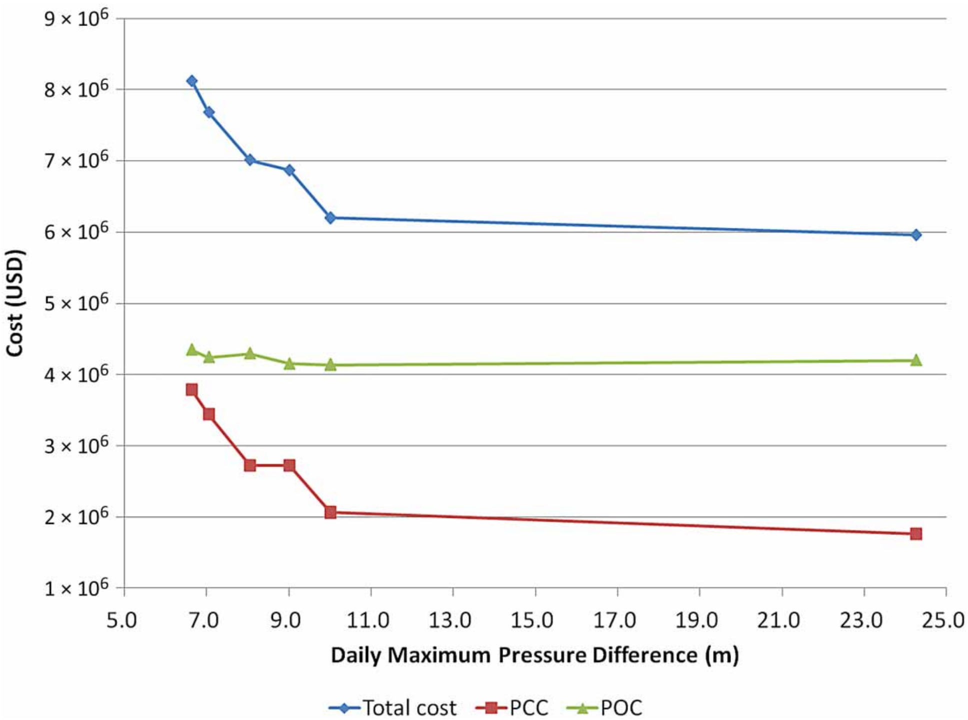

Figure 5 | Total cost and daily maximum pressure difference of Pareto optimal solutions for pump optimization of the Apulian-35 network.

Table 3 | Austin network's pump design and scheduling solutions with the original tank capacity (tank volume = 3,779 tons)

Daily maximum pressure difference (m)	Pump capacity (kW)	Number of pumps	POC (%)	PCC (%)	Total cost (USD)
24.3	75	2	4,200,470 (71)	1,756,790 (29)	5,957,260
10.0	55	3	4,136,083 (67)	2,061,125 (33)	6,197,208
9.0	45	5	4,149,892 (60)	2,719,809 (40)	6,869,701
8.0	45	5	4,291,177 (61)	2,719,808 (39)	7,010,985
7.0	45	7	4,240,651 (55)	3,442,135 (45)	7,682,786
6.6	45	8	4,342,599 (53)	3,779,396 (47)	8,121,995

Table 4 | Austin network's pump design and scheduling solutions obtained with a bigger tank capacity (tank volume = 12,755 tons)

Daily maximum pressure difference (m)	Pump capacity (kW)	Number of pumps	POC (%)	PCC (%)	Total cost (USD)
26.9	90	2	3,593,519 (66)	1,889,698 (34)	5,483,217
15.0	75	2	3,923,947 (69)	1,756,790 (31)	5,680,737
10.0	55	3	4,006,493 (66)	2,061,125 (34)	6,067,618
9.0	55	3	4,416,814 (68)	2,061,125 (32)	6,477,939

variation in the head gain. As a result, the daily maximum pressure differences were smaller than the Apulian networks (Table 5). Similarly, smaller pumps were constructed, which lowered the total cost relative to the Apulian networks. While the POCs were similar among the solutions, the PCC increased when a high robustness level was considered.

Table 5 | Net2 network's pump design and scheduling solutions

Daily maximum pressure difference (m)	Pump capacity (kW)	Number of pumps	POC (%)	PCC (%)	Total cost (USD)
2.51	2.5	2	175,881 (19)	754,140 (81)	930,020
1.0	1.4	3	182,646 (17)	864,640 (83)	1,047,286
0.7	0.9	5	185,566 (14)	1,118,077 (86)	1,303,643

Pump design and scheduling results

In order to enhance the system operational robustness, the number of pumping units gradually increased as the pump capacity decreased (Tables 3–5). For example, in the Apulian-35 network, the least-cost solution (daily maximum pressure difference = 24.3 m) had two identical pumping units of 75 kW, while the most robust solution (daily maximum pressure difference = 6.6 m) constructed eight pumping units of 45 kW. The most robust solution in the Net2 network constructed five identical pumping units of 0.9 kW, while the least-cost solution (daily maximum pressure difference = 2.51 m) had two pumping units of 2.5 kW. This was to diversify the head gains from the pump station, which made it possible to bound pressure variations within a narrow range.

This result quantitatively confirmed Lansey's (2012) suggestion of using multiple pumping units in order to enhance robustness and redundancy (i.e., the two components of WDS resilience). The results of this study led to conclusions similar to those of Zhuang et al. (2013), who proved that having multiple pumping units and an adaptive pump operation scheme helps improve the water availability of a WDS under failure conditions.

Figures 6 and 7 show the 24 h optimal pump schedule determined according to the four constraints (24.3, 10, 7, and 6.6 m) of the Apulian-35 network and the three constraints (2.51, 1, and 0.7 m) of the Net2 network. Because the total number of pumping units increased as the operational robustness level decreased, the number of available pumping units increased (Figures 6 and 7). For example, Figure 6(a) shows the pump schedules of two 75 kW pumps, while Figure 6(b)–6(d) show those of three 55 kW pumps, seven 45 kW pumps, and eight 45 kW pumps, respectively. Note that the pump capacity decreased at

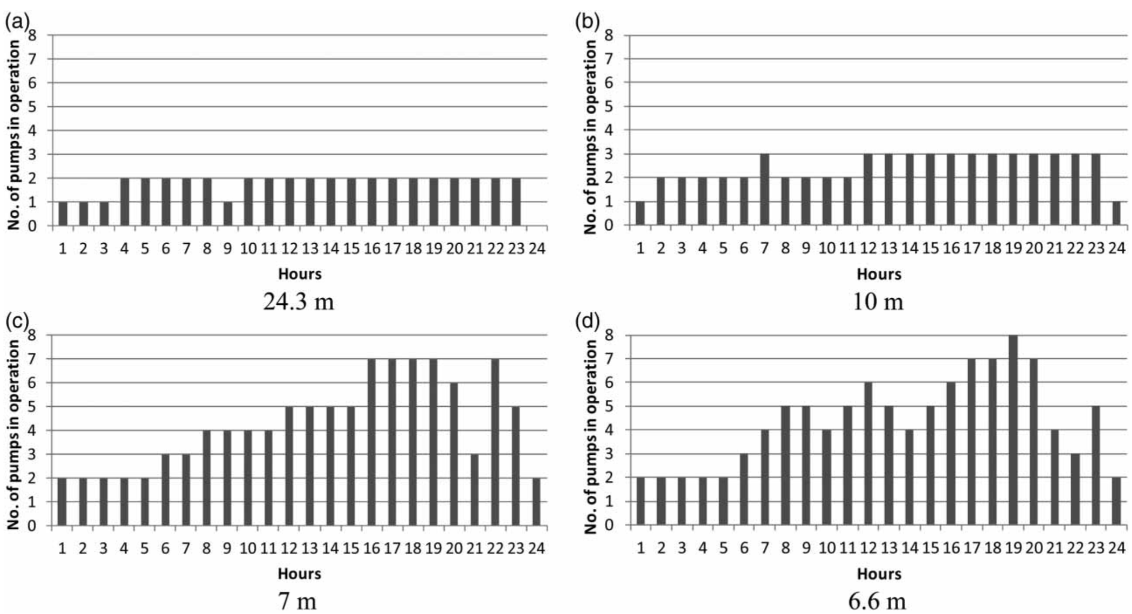

Figure 6 | Comparison of pump schedules determined for the Apulian-35 network: (a) least-cost scheduling, (b) $SRob_{req} = 10$ m, (c) $SRob_{req} = 7$ m, (d) $SRob_{req} = 6.6$ m.

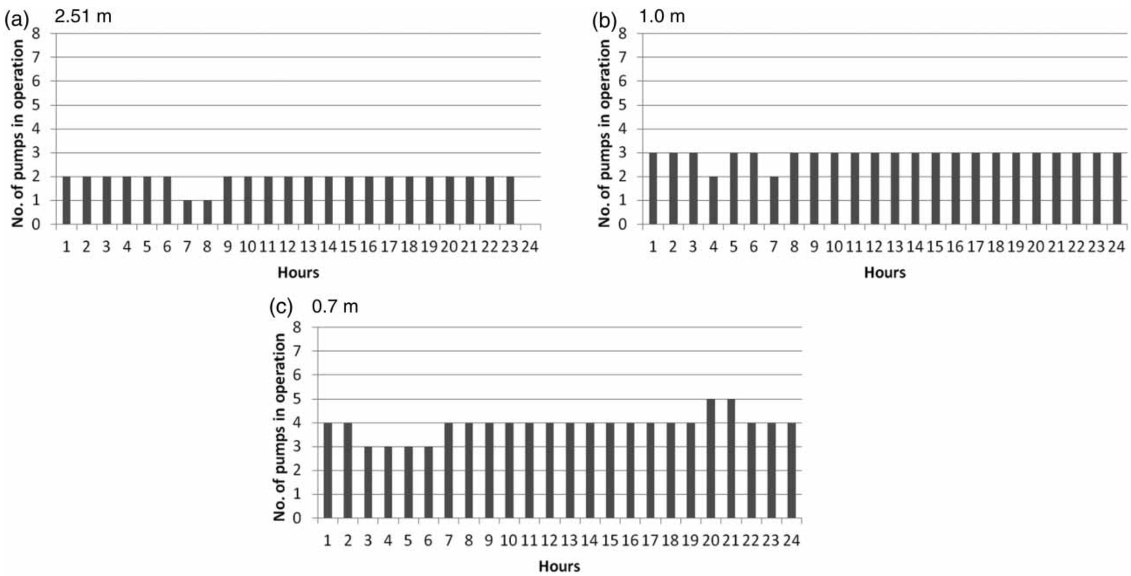

Figure 7 | Comparison of pump schedules determined for the Net2 network: (a) least-cost scheduling, (b) $SRob_{req} = 1.0$ m, (c) $SRob_{req} = 0.7$ m.

medium robustness levels and became constant at high robustness levels in the Apulian-35 network, while it consistently decreased in the Net2 network.

As expected, we observed more pumping around the peak hours, especially in the evening (6–9 pm) (see Figures 6(c), 6(d), and 7(c)). This was the result of the considered demand pattern for which the center of mass lay around the evening hours (Figure 4). The pump scheduling pattern of 6.6 m (Figure 6(d)) of the Apulian-35 network solutions resembled the demand pattern the most.

Compared with the other cases, the pump scheduling when operational robustness was not considered showed relatively constant pumping (see Figures 6(a) and 7(a)). For example, in the Apulian-35 network, the two pumping units were operational except at 12–3 am, 8–9 am, and 11 pm–12 am. Note that pressure variations were not limited in this case. In this solution, the two pumps operated even under low demand conditions, which resulted in the water level rising in the tank. For example, the operation of the two pumps at 3–8 am raised the tank water level from 59 to 60 m (maximum tank level) (Figures 6(a) and 8(a)). The stored water volume during the period was used for the morning peak hours (7 am–12 pm) to reduce pump energy costs by minimizing pumping during the peak hours (Figure 6(a)). The demand factors during the period (3–8 am) were mostly between 0.58 and 0.98, while the

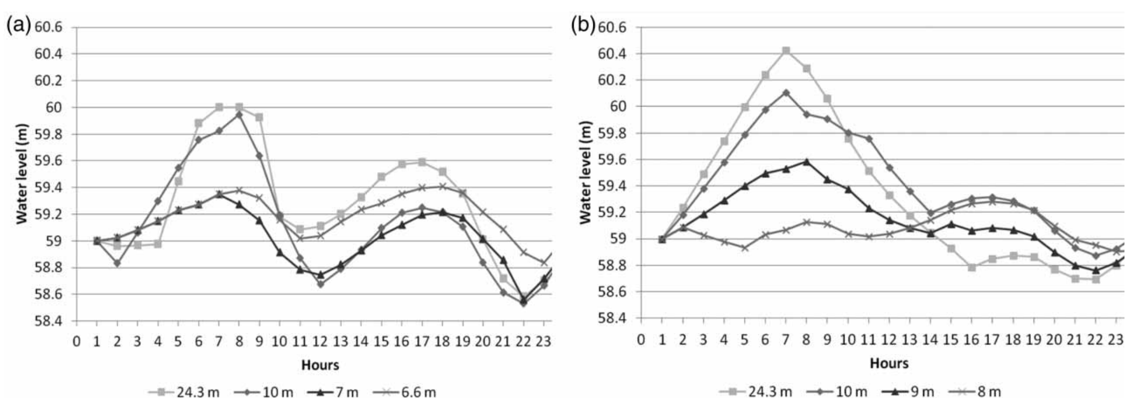

Figure 8 | Tank water level trajectories for 24 h according to two sets of schedules: (a) for the Apulian-35 network and (b) for the Apulian-52.5 network.

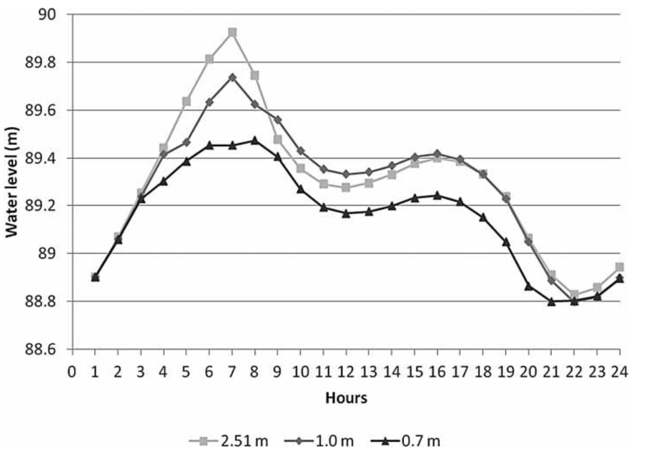

Figure 9 | Net2 network tank water level trajectories for 24 h according to schedules in Figure 7.

unit energy cost was 0.08 USD/kWh. On the other hand, the rise in the water level in the tank in the late afternoon (12–5 pm) for the evening peak hours was not as significant as that in the morning because of the high demand (demand factors were between 0.98 and 1.2) and high energy tariff (0.17 USD/kWh) for pumping during the afternoon off-peak hours. This resulted in an increase of less than 0.5 m for the water level in the tank. Similar results were obtained for the Apulian-52.5 and Net2 networks (Figures 7(a) and 9).

Tank trajectory comparisons

The aforementioned traditional patterns of tank levels were common to all of the solutions regardless of the study network. However, the tank level variations were bounded in solutions obtained with a high robustness level. Because the pressures within the system are also a function of the water level in the tank, the tank water levels were bounded because of the constraint on the pressure variations. For example, in the Apulian-35 network, the tank water levels of the high robustness solutions (7 and 6.6 m) varied between 58.5 and 59.4 m (0.9 m difference), while that of the least-cost solution varied between 58.5 and 60 m (1.5 m difference) (Figure 8(a)). In the Net2 network, the tank water level of the high robustness solution (0.7 m) varied between 88.9 and 89.5 m (0.6 m difference), while that of the least-cost solution varied between 88.9 and 89.9 m (1 m difference) (Figure 9). In other words, the high-robustness solution provided little benefit when the stored tank water was utilized during peak hours.

Comparing tank trajectories of two Apulian networks (Figure 8(a) and 8(b)), the Apulian network with a bigger tank capacity allowed large variations in the tank trajectories (Figure 8(b)). In order to fill a large volume of water within a limited time (i.e., to fully utilize the storage capacity of a big tank), the Apulian-52.5 network required a bigger pump than the Apulian-35 network (e.g., two 90 kW pumps in the former and two 75 kW pumps in the latter). Finally, all of the solutions satisfied the constraint on the water level in the final time period, which was considered to ensure continuity in the pump scheduling.

Pressure variation comparisons

Figure 10(a) and 10(b) show the pressure changes at node 10 (dashed circle in Figure 1), which was one of the most critical nodes, over 24 h with the pump schedules determined by

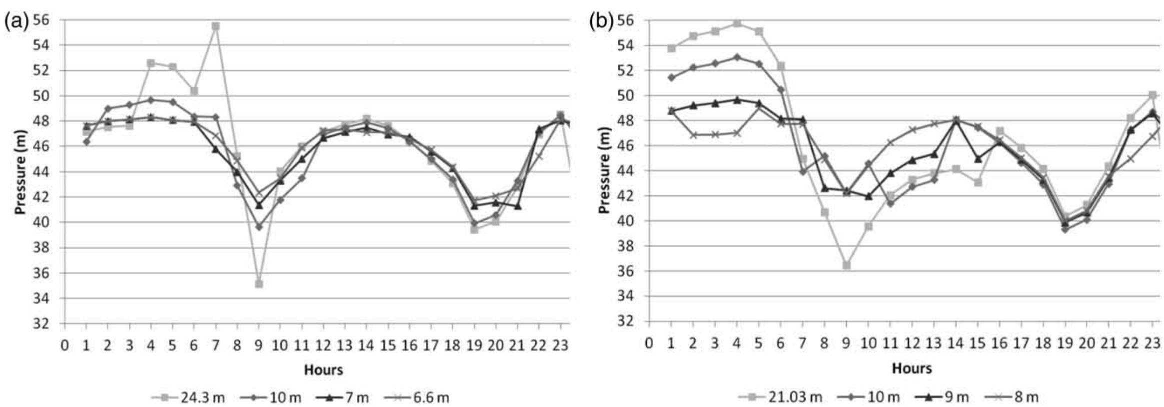

Figure 10 | Pressure changes at node 10 in the Apulian networks over 24 h according to the schedules in Figure 6.

using the Apulian-35 and 52.5 networks, respectively. Note that the other critical nodes were nodes 1, 12, and 13 (Figure 1). The least-cost solution was not robust because there was a large pressure decrease from 55.5 to 35 m (20.5 m or 29.2 psi decrease) within 2 h in the Apulian-35 network and 56 to 36 m (20 m or 28.4 psi decrease) within 5 h in the Apulian-52.5 network. This was the result of turning off a 75 kW pump at 8 am (peak hour) in the former (Figure 6(a)) and turning off a 90 kW pump from 6 am to 3 pm in the latter (pump schedule not presented) to reduce the pump energy cost. Similarly, in the Net2 network, the least-cost solution had the largest variations in pressure at the critical node (node 1 marked as dashed circle in Figure 2) with a sudden pressure drop of 1.5 m around midnight.

Sudden pressure changes within the system can cause low serviceability and thus should be avoided. In practice, system operators do not operate the pumps in this manner. As a result, traditional pump scheduling approaches that only consider the POC have not received a great deal of attention. In all study networks, changing the status of the pumps did not result in significant changes in the pressure with the high-robustness solution (compared to the least-cost solution) because small pumps were used (Figures 10 and 11). It was confirmed that constructing many small pumps is beneficial to achieving robust system operation.

The proposed robustness-based pump design and operation can constrain pressure variations to within the desired level while minimizing the total pump cost and satisfying other constraints on the pressure requirements and tank levels. With the diurnal pressure pattern (low pressure around peak hours), the daily maximum pressure difference was limited to predefined levels (Figures 10 and 11). The proposed pump design and operation scheme was proved to be useful at not only minimizing the total pump cost but also controlling both the system pressure and the tank's water level.

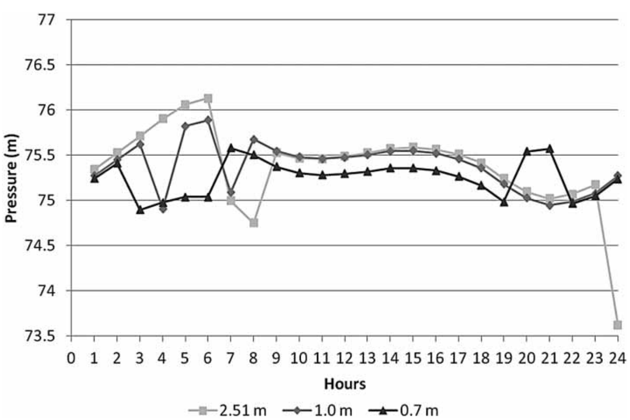

Figure 11 | Pressure changes at node 1 in the Net2 network over 24 h according to the schedules in Figure 7.

SUMMARY AND CONCLUSIONS

While various types of system robustness and reliability have been adopted for optimizing the pipe sizes of a water distribution network, little effort has been made towards considering system performance measures in optimizing the pump design and operation. However, reliability and robustness (e.g., maintaining the system pressure above certain level) are the most important factors considered by water distribution operators.

This study was the first to develop a system operational robustness indicator and introduce the proposed indicator to water distribution network pump design and operation. The proposed constrained optimization model minimizes the pump construction and operations cost with constraints on the system robustness, pressure requirement, and tank level. Here, robustness was defined as a system's ability to maintain its pressure within a given range in the event of disturbances to the demand and was calculated as the daily maximum pressure difference at the critical node. The Apulian and Net2 networks were employed to demonstrate the use of the proposed method to design the number of pumps and their schedule over 24 h. The Pareto relationship between the total pump cost and system robustness was investigated by optimizing the proposed model with robustness constraints of 10, 9, 7, and 6.6 m for the Apulian network (Apulian-35) and 1.0 and 0.7 m for the Net2 network. In addition, similar optimization was conducted for an Apulian network with a bigger tank (Apulian-52.5) to investigate the impact of the tank size on the pump design and operation. Each optimization was performed independently. The resulting pump designs and schedules were compared with those obtained from the traditional least-cost decision-making.

Compared to the least-cost operation, regardless of the study network, the robustness-based operation resulted in consistent pressure throughout the day (i.e., the desired pressure difference was achieved). In order to achieve high operational robustness, a larger number of small pumps were constructed and operated. The water level in the tank varied within the bounded range, indicating that the robust operation scheme limited not only the system pressures but also the water level.

Improved resourcefulness from the increased tank capacity helped improve the cost-effectiveness of the robustness-based pump design and operation to achieve the same level of robustness. To compensate for the bigger tank size, a bigger pump was constructed for the Apulian-52.5 network than for the Apulian-35 network. Comparing the tank trajectories of the two Apulian networks, the Apulian-52.5 network allowed large variations in the tank trajectories.

The results of this study have several limitations that future research should address. First, variable-speed pumps (VSPs) can be considered instead of fixed-speed pumps for robustness-based pump design and scheduling. For fair comparison of the cost efficiency, an accurate PCC function should be provided for VSPs. This study formulated a constrained pump design and operation problem. However, this problem can be solved by multi-objective optimization to explore the full profile of the Pareto optimal solutions. Fire flow and pipe burst conditions can be considered in the pump optimization. Identifying the relationship between the level of robustness and the magnitude of system leakage is an interesting research topic.

In addition, the tank capacity (i.e., dimensions) can be considered as a design parameter in order to explore the interdependence among three system design parameters (i.e., pump capacity, pump operation, and tank capacity). Finally, a comprehensive water distribution network design problem can be formulated to consider the pipe sizing, pump and tank design, and pump scheduling simultaneously.

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